NATIONAL BUREAU OF STANDARDS REPORT

R 7400879

6530

TRANSLATION
by
Bernard H. Fouquet
of paper in french

METHODS ADOPTED IN THE U.S.S.R. FOR THE STUDY OF FIRE RESISTANCE OF BUILDING ELEMENTS AND SOME OF THE RESULTS OBTAINED

by

V. I. Mourachev

Active member of the Academy of Building and Architecture of the U.S.S.R., Doctor in Science and Professor.

NBS

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

THE NATIONAL BUREAU OF STANDARDS

Functions and Activities

The functions of the National Bureau of Standards are set forth in the Act of Congress, March 3, 1901, as amended by Congress in Public Law 619, 1950. These include the development and maintenance of the national standards of measurement and the provision of means and methods for making measurements consistent with these standards; the determination of physical constants and properties of materials; the development of methods and instruments for testing materials, devices, and structures; advisory services to Government Agencies on scientific and technical problems; invention and development of devices to serve special needs of the Government; and the development of standard practices, codes, and specifications. The work includes basic and applied research, development, engineering, instrumentation, testing, evaluation, calibration services, and various consultation and information services. A major portion of the Bureau's work is performed for other Government Agencies, particularly the Department of Defense and the Atomic Energy Commission. The scope of activities is suggested by the listing of divisions and sections on the inside of the back cover.

Reports and Publications

The results of the Bureau's work take the form of either actual equipment and devices or published papers and reports. Reports are issued to the sponsoring agency of a particular project or program. Published papers appear either in the Bureau's own series of publications or in the journals of professional and scientific societies. The Bureau itself publishes three monthly periodicals, available from the Government Printing Office: The Journal of Research, which presents complete papers reporting technical investigations; the Technical News Bulletin, which presents summary and preliminary reports on work in progress; and Basic Radio Propagation Predictions, which provides data for determining the best frequencies to use for radio communications throughout the world. There are also five series of nonperiodical publications: The Applied Mathematics Series, Circulars, Handbooks, Building Materials and Structures Reports, and Miscellaneous Publications.

Information on the Bureau's publications can be found in NBS Circular 460, Publications of the National Bureau of Standards ($1.25) and its Supplement ($0.75), available from the Superintendent of Documents, Government Printing Office, Washington 25, D. C.

Inquiries regarding the Bureau's reports should be addressed to the Office of Technical Information, National Bureau of Standards, Washington 25, D. C.

NATIONAL BUREAU OF STANDARDS REPORT

NBS PROJECT **NBS REPORT**

1002-12-10120 September 1, 1959 6530

TRANSLATION
by
Bernard H. Fouquet
of paper in french

METHODS ADOPTED IN THE U.S.S.R. FOR THE STUDY OF FIRE RESISTANCE OF BUILDING ELEMENTS AND SOME OF THE RESULTS OBTAINED

by

V. I. Mourachev

Active member of the Academy of Building and Architecture of the U.S.S.R., Doctor in Science and Professor.

NBS

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

Original: presented in
French by
M. Mourachev

METHODS ADOPTED IN THE U.S.S.R. FOR THE STUDY OF FIRE RESISTANCE OF BUILDING ELEMENTS AND SOME OF THE RESULTS OBTAINED

by

V. I. MOURACHEV

Active member of the Academy of Building and Architecture of the U.S.S.R., Doctor in Science and Professor.

1. General principles on the evaluation of fire resistance.

The elaboration of a method for the correct evaluation of the fire resistance of structural elements, particularly elements in reinforced concrete, has acquired a considerable importance.

There are two contributing factors:

First, reinforced concrete occupies a preponderant situation in buildings; second, structures of reinforced concrete are extremely varied. The use of thin-walled elements, the utilization of metals of high elastic limit and in particular prestressed cold-drawn steel wires HR, have a marked influence on the technical properties of the construction, fire resistance included.

The answer to questions relative to fire resistance of elements used in structures, and of entire buildings, cannot be based only on trials of these elements and buildings. The intensity of the entire fire must be studied.

The notion of the limit of resistance to fire which is measured in hours and that of the theoretical duration of the fire also expressed in hours must be defined. These two values are related by a coefficient that we shall call the index of fire resistance. It is in multiplying the theoretical duration of the fire by this index that we obtain the value of the limit of resistance to fire required. The real value of the resistance of an element cannot be less than the required value. One thus obtains the expression* used for the computation of the fire resistance:

$$R_r \geqslant K_o D \qquad (1)$$

where:

R_r is the real limit of resistance to fire of an element expressed in hours.

* V. I. Maurochev. Resistance of structural elements to thermal exposures.

D is the theoretical duration of the fire.

K_o is the index of fire resistance.

The performance requirements to which the elements are subjected are a function of the fire duration "D", the latter depending on the type and on the quantity of combustible materials and on its physical state, form and dimensions of the rooms as well as the presence of automatic extinction equipment and other factors.

Relationship (1) makes possible the standardization of the limit of fire resistance of the elements of construction for a given duration of fire by introducing standardized values for the index of resistance.

The value of this index for the entire building is chosen according to the degree of non-combustibility desired, that is to say, according to its behavior required during and after the fire. With regard to the different elements, the choice of values for the indices is determined by their relative importance and by the importance of the equipment and its various positions in the building, etc.

The value of K_o can be below or above unity in accordance to the degree of the resistance to fire required of the building and of its various parts and elements.

In this manner relationship (1) furnishes the necessary data in the computation of the fire resistance of the buildings and other structures.

The experimental and theoretical research undertaken at the Central Institute of Fire Fighting under the direction of the author is based on the fundamental considerations given above. These considerations, however insufficient they may be, form the basis for the Soviet standard related to fire fighting. (Standard 102-54)

Present research is directed to studying the limit of resistance to fire of building elements as well as defining more precisely this theory in itself. On the other hand this research covers also the intensity of the fire D as a function of the different types of combustibles and the varied conditions of their combustion.

The intensity of a fire is characterized by its duration and by the temperature developed during the fire. During the combustion of wood and other similar materials (paper, cloth etc.) the temperature-time curve of a fire goes through three general well defined phases (fig. 1):

Phase I - period of ignition during which the temperature in the module rises in undefined fashion,

Phase II - period of intense combustion with rapid rise in temperature until maximum is obtained,

Phase III- period of extinction of the fire with a drop in temperature at first rapid and then considerably slower.

It would be rather inconvenient to use these curves in an experimental determination of fire resistance of buildings as well as for the evaluation of the duration of the fire.

For this reason, a standard time temperature curve of a theoretical fire (fig. 2) has been adopted which closely resembles the standard time-temperature curves adopted by other countries.

In Fig. 2 we have included the standard curve with time temperature curves for real fires obtained by burning 25, 50, 100 and 150 kg. of wood to a square meter of floor of a module. The parts of these curves related to the ignition period which have no influence on the rise in temperature of the building have been omitted. The conversion of the duration of a real fire to the duration of a standard fire is performed in the following manner: For the duration of the theoretical fire the time for which the area under the standard curve equals the corresponding area under the real curve has been adopted; (see fig. 3) this duration is plotted along the abscissa and is expressed in hours.

It can be seen that the rise in temperature of the surrounding walls in fires that follow the standard curve differ very slightly from the rise in temperature produced during a real fire.

In obtaining the conversion, the ignition period is neglected, and for the terminations of the real fire we chose from the time temperature curve of the latter the point at which the real heating begins to express a slower rate than the theoretical heating. Experiments conducted at the Institute show that this takes place when the temperature of the real fire lowers to 600° to 400°, fig. 3.

In taking this data as a basis, it can be determined with enough precision that the duration of the theoretical fire in a residential building can be obtained in the following manner (assuming the absence of all extinguishers)

$$D = \frac{T \quad q}{200{,}000} = 0.02\ q \qquad (2)$$

where: T is the specific heat of wood and similar materials equal, on the average, to 4,000 Kcal/kg

q is average quantity of combustibles in kg per square meter of floor space.

200,000 is the conventional quantity of heat in Kcal emitted in one hr per square meter of an area.

In the most general case, the presence of various liquid and solid combustibles in the room, the expression representing the theoretical duration of the fire is given in the following form:

$$D = f\ (q,\ T, \beta, \gamma, \alpha,\ c) \qquad (3)$$

where:

q is the average quantity of each type of combustible in kg of square meter of floor

T is specific heat in Kcal/kg

β is the coefficient of combustion of each material which depends on its proximity to the fire.

C is the conventional quantity of heat in Kcal emitted in one hour and per square meter of floor space.

γ is the factor taking into account the shape, the dimensions, and the degree of ventilation of the considered space.

α is the factor taking into account the influence of the active extinguishing methods taken to put out the fire, such as: automatic equipment, firemen in the vicinity and access to the building.

The real limiting resistance of the structure is determined experimentally under a thermal system analagous to that of the theoretical fire (standard). For a number of elements the Institute has evaluated methods permitting the determination of this limiting resistance by means of computations. Whatever may be the case a precise definition of the terms must be found for the limiting resistance to fire of constructions as a function of the services that they may furnish ordinarily and in times of fire and one must enumerate the circumstances denoting that this limit has been attained.

It is generally conceded that the limiting resistance which is expressed in hours is exceeded when the element in question ceases to be fire retardant or loses its quality of mechanical strength during and after the fire. Up to then the signs characterizing the limit mentioned are the following:

1) The appearances of breaks through which flames can pass into the adjoining room and can then start a fire;

2) The rise in temperature on the cold side of a wall above the maximum temperature on the cold side of a wall above the maximum temperature established by standards;

3) Loss of the supporting power of the element.

The two first signs mentioned are sufficiently clear and are of importance only during the actual fire.

The third sign requires to be studied more thoroughly, the supporting power of an element being of extreme importance not only during the fire but after.

2. <u>Behavior of building elements during a fire and criterion of the limiting state of its supporting power</u>.

In modern buildings it has been demonstrated in experiments at the Institute that resistance differs according to the action of a fire. It follows that a precise definition universally adopted for the limiting state of a building element's supporting strength acquires great importance.

The criterion of complete destruction (collapse) of an element cannot be adopted because it does not take into account its state preceding the immediate failure and in particular the irreversible changes and the loss of the supporting strength. This quality is of extreme importance for old buildings computed with K_o greater than 1, that is, when the supporting strength and the rigidity must be retained during and after the fire.

In other terms, if we adopted the criterion of total destruction (collapse), the structural elements developed differently but possessing the same limiting resistance to fire and having been submitted to the same type of fire would not retain the same supporting strength and would not guarantee the same security during and after the fire. The time elapsed to total destruction is significant only for elements calculated with K_o smaller than 1, that is to say, for those which must only stand a certain number of hours during the fire without being of any importance after the end of the fire.

Consequently, we shall adopt as the criterion of limiting fire resistance, with regard to supporting strength and rigidity of an element, the increase in irreversible changes to the point where they become unusable.

In order to explain the principles given above, we will use the results of the experimental studies of fire resistance of elements in, reinforced concrete (mild steel) and prestressed concrete, (steel wire HR cold-drawn.)

In our experiments, the destruction of these two types of elements by the action of fire started by the formation of a plastic failure (points O on fig. 4), that is to say, when because of the rise in temperature the limit of the elasticity of the mild steel and the resistance of the steel wires HR decreases to the value of the forces exerted during the normal burning. At that time there are observed the opening of large breaks in the zone of stress and a rapid increase in the deformation. In fig. 4 there are reported the values for total changes of each element composed of deflections caused by the variation in temperatures in the section of those elements, (reversible changes) and supplementary deflections caused by the modification of the physical and mechanical properties of the concrete and the steels by the action of the heat. This last deformation is irreversible and influences considerably the evaluation of limiting resistance to fire. With regard to supporting strength, since the larger residual deformations always occur simultaneously with a loss of supporting strength, such deformations signify that the limit of supporting has been reached.

In order to compute this residual deformation, experiments were undertaken during which the rise in temperatures of the elements were stopped slightly before the formation of the plastic failure. After the flames were extinguished, the elements still under load were left to cool slowly at the same rate as the oven. The experiments were conducted on the reinforced concretes with mild steel and steel wire HR prestressed.

The curve 1 of the fig. 5 represents the increase in total deflection of one of the four beams tested, reinforced with mild steel (steel No. 3) on which the heating was conducted for 49 minutes and stopped three minutes before the point of the plastic failure. The decrease in deflections is illustrated by the curve No. 2 which is parallel to curve No. 3, the latter represents the deformation due only to heat. If curve No. 2 is displaced to point "a" it will coincide practically with curve 3.

The difference between the total deflections and the changes due to heat alone furnishes the value of the residual deflections under load due to the modifications of the physical and mechanical properties of the materials caused by heat. These modifications are explained by the hot flow of the metal and of the concrete and by a change in the elastic properties of the concrete.

In examining the curves represented in fig. 5 it is seen that for an element reinforced with ordinary steel, the irreversible changes constitute only a small percentage of the total deflections up to the point where the plastic failure occurs.

Consequently, we cannot adopt as limiting resistance with regard to supporting strength of the elements in reinforced concrete with mild steel, the time corresponding to the formation of the plastic failure (point O) which immediately precedes the total collapse.

The curves represented in fig. 6 furnish a completely different picture. In this figure we have plotted the total changes and the reversible deformations of three panels reinforced with prestressed wires of ultimate strength of 18,000 kg/cm^2,* the first wall being heated to 313° (42 min.), the second panel to 315° (36 min.) and the third panel to 375° (45 min.), after which the three panels were cooled slowly under load.

* 1 kg/cm^2 = 14.22 lb/in.^{2} (added in translation)

It can be judged from these curves that the irreversible change of the elements reinforced with prestressed steel wires HR (difference between curves 1 and 2; 3) constitute approximately 60% of the total changes and are developed as soon as the heating of the metal occurs.

The beginning of the residual deflections are explained by the hot flow of the metal submitted to very high stresses. This reasoning is confirmed by preliminary experiments conducted at the Institute on the hot flow of steel wire HR of 15,000 kg/cm^2 and heated to 300°, with subsequent cooling. Even at 200° the flow becomes very apparent and attains a considerable value at a temperature of 300°. The rate of heating of the wires is similar to the heating of the reinforcements in panels 1 and 2. After complete cooling the residual deformation due to flow, per unit length, was equal to .0038. Also the similar deformation leads to a complete loss of pretension and to the formation of important irreversible changes of deflection. In effect, for the ribbed panels which have been used in the experiment described above, an elongation of the steel equal to .0038 would result in a deflection amounting to 1% of the span.

Experiments conducted on the elements reinforced with steel of ultimate strength equal to 16,000 kg/cm^2 have confirmed these conclusions. The heating of steel to 300° resulted in a total loss of pretension and in a reduction of rigidity to less than 1/3 of the initial rigidity.

Prestressed beams, after heating and cooling, behaved under load in the same manner as non-prestressed beams.

Upon heating of the reinforcements to 220° a loss of pretension equal to 50 to 55% and a loss in rigidity of the beams equal to 1/2 resulted.

As has already been said, one indication of the limit to fire resistance in these elements is the formation of undue residual deflections. In fig. 7 we have then the curves of irreversible changes on the elements tested as well as the temperature of their reinforcements.

It can be noted that the residual deformations of the beams reinforced with mild steel (curve A) increase in time proportionately to the heating of the metal to the point of deformation of the plastic failure; that is to say to the time when, under the heat, the limit of elasticity of the steel falls to a value equal to the existing stresses.

In the case considered the steel had elastic limit of 2,340 kg/cm^2 which decreased to 1,460 kg/cm^2 (value equal to the working stress) at a temperature of 460°. At that time the residual deflection was only $\frac{1}{180}$ of the span.

It can then be seen that the protection of this type of construction during and after the fire can be assured, if that is required by the existing standards, by adopting as an index of fire resistance the values of 1.1 to 1.2. In this case the residual deflections cannot be any higher than $\frac{1}{230}$ to $\frac{1}{280}$ of the total span and the supporting strength of these elements after the fire would be reestablished completely because after cooling the soft steel regains all of its mechanical resistance. After the fire, minor repairs on the lower surface of the elements will be sufficient.

However, the deformations of the elements reinforced with prestressed wires of HR steel curve 1, 2 and 3 of fig. 7 will reach, at the time of the plastic failure, (450° in our case) the much higher values of 1/50 to 1/40 of the span. This signifies that the elements become unusable for other reasons much before the limit of resistance of the steel has reached the existing stresses.

As we have indicated previously this comes from the loss of pretension and of the hot flow of the metal. It follows that for the prestressed concrete the elapsed time before the appearance of the plastic failure cannot be adopted as a measure of the limiting resistance to fire because this does not guarantee the salvaging of the elements in question after the fire if this is required by the standards.

However, for buildings of the third and fourth categories of non-combustibility (Soviet standard) for which the salvaging of the elements after a fire is not required, only the time of heating before complete destruction is important in the evaluation of the fire resistance of those elements.

It can be seen then that the appearance of irreversible changes is a sign which indicates that the limiting resistance has been attained. However, there are other factors which must equally be taken into consideration. Thus the limitation of residual deflections can guarantee security only for the buildings which after cooling regain their supporting strength and their rigidity. We have already shown that this is the case for concrete reinforced with ordinary mild steel.

For concrete reinforced with steel wire HR, the element will regain its initial supporting power only if the maximum temperature obtained during the fire is not sufficient to provoke the irreversible loss of strength due to cold working.

Experiments indicate that for steel of 18,000-20,000 kg/cm^2 such a loss of strength becomes noticeable at 200^o. Fig. 8 shows clearly that this loss is a function not only of the relative value of the increase in strength obtained by cold working but also of the thermal exposure.

A rise in temperature to 800^o produces in the steel a complete loss of resistance and reduces it to its primitive state.

Thus the loss in supporting strength constitutes the second factor which shows that the limiting resistance has been obtained.

Also, the irreversible loss of rigidity indicates that the element has reached its limiting resistance. For reinforced concrete such a loss comes from the lowering of modulus of the concrete after heating, from a partial loss of the bond between the concrete and the steel, and in the case of prestressed concrete, in the loss of pre-tension.

Actually, the standards in existence in the U.S.S.R. relative to structures made with reinforced concrete take into consideration that the limit of resistance of an element is attained in the following cases:

1. The irreversible changes (residual deflection), allowance being made for an initial working deflection, reaches 0.01 of the free span.

2. An irreversible loss of supporting power of 10%.

3. An irreversible loss of rigid strength of 20%.

Fig. 7 shows that prestressed elements reinforced with wires of HR steel of 18,000 kg/cm^2 reach a residual deflection equal to 0.01 of the span when the metal is heated to 200-300°. At 200° we observe an irreversible loss in rigidity and in the resistance of the steel. (see Fig. 8)

It is evident that the admissible temperature to which HR steel can be subjected depends on the relative value of the supplementary resistance obtained when the wire is cold-drawn. But the experimental data relative to elements reinforced with steel, the resistance of which is below 15,000 kg/cm^2, is still lacking.

Analysis of data furnished in fig. 7 indicates that at the thickness equal to the protective thickness when the rate of heating of the metal is the same, the limit of resistance to fire of the supported elements reinforced with mild steel is at least twice that of the elements reinforced with steel wires HR of 15,000 to 18,000 kg/cm^2. That is the reason the standards (Standard 102-54) require, for the element reinforced with wires HR, corresponding increase in the thickness of the cover.

The experiments conducted at the Institute show also that the limiting resistance of the elements incased in supports (or in the presence of adequate restraint) increases considerably if the relation between the sections of steel at the supports and in the span is not less than 2. The time elapsed before collapse increases in the ratio of 1 to 3 (see curve 3 of fig. 4) because of redistribution of stresses which are produced during the heating making it possible to achieve a temperature of 800-900° in the longitudinal reinforcement.

As for elements reinforced with steel wires HR of 15,000-18,000 kg/cm^2 the limiting resistance is not influenced by an incasement or by the presence of a restraint because the heating of the metal in the central sections above 300° cannot be tolerated.

Concerning elements reinforced with welded reinforcements of soft steel worked in the cold (6,000 kg/cm^2) the limit of resistance does not differ from that of elements reinforced with ordinary mild steel.

The irreversible decrease in resistance for this type of reinforcement begins to manifest itself (above 5%) only when a temperature of 450° is reached, (fig. 8) namely, shortly before the finish of the plastic failure. The residual deformations are also relatively small compared to the total deflections because the working stress for this type of steel (2,800 kg/cm^2) is not very high. Fig. 9 shows the total deflections and residual deflections of a ribbed panel with welded reinforcement of cold drawn steel wires. Heating the wires up to 490° formed an irreversible deflection of 13 mm which is not more than 1/240 of its span.

The use of deformed steel does not influence the limit of resistance to fire of the concrete because an irreversible loss of resistance of this steel does not begin to show itself before 500° (7%) namely at the time of the formation of the break.

The experiments of the Institute have also shown that elements reinforced with special hot rolled steel "25 GS" (limit of elasticity 4,000 kg/cm^2) have limit of resistance to fire slightly above those of constructions with reinforcements of mild ordinary steel (steel No. 3) or with welded mesh of steel wire of 6,000 kg/cm^2.

The formation of the plastic break did not appear until 570°. and the residual deflections just before the finish of this break did not exceed 0.01 of the span and no irreversible loss of supporting power was discerned.

CONCLUSIONS

1. The limit of resistance to fire of simply supported reinforced concrete elements is attained at the moment of the plastic failure (when the limit of the elasticity of the pre-stressed steel is attained). Just before the formation of this break irreversible deflections do not exceed 0,01 of span and the irreversible losses in supporting strength and rigidity remain within the limits of toleration.

2. The limit of resistance to fire of prestressed elements, also simply supported, reinforced with steel of 15,000-18,000 kg/cm^2 strength, is approached when the heating of the metal

reaches 200-300°. Above 300° the irreversible loss in supporting strength begins to increase at very rapid rates, whereas at temperatures slightly above 200° the irreversible losses in rigidity exceed the toleration affixed at 20%. For the cold drawn steel of 8,000-12,000 kg/cm^2, the maximum heating tolerated must be determined by further experimentation.

3. For elements with ordinary reinforcements, restraint at the supports permits the increase of limiting resistance if the relation between the effective section of steel at the supports and on the span is not inferior to 2. For prestressed elements reinforced with steel HR restraint at the supports does not produce an increase in the limit of resistance.

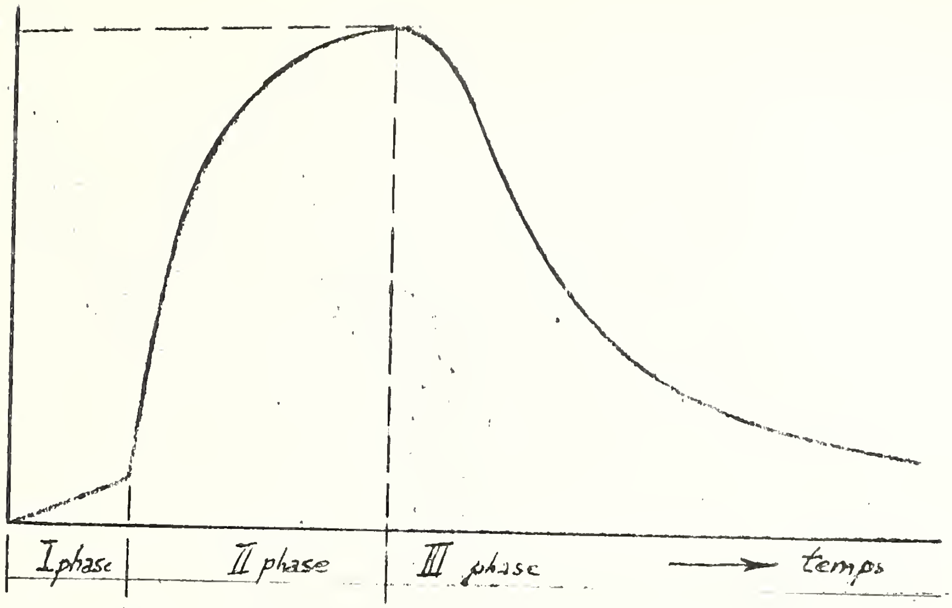

Figure 1. Characteristic temperature-time curve for a fire involving wood.

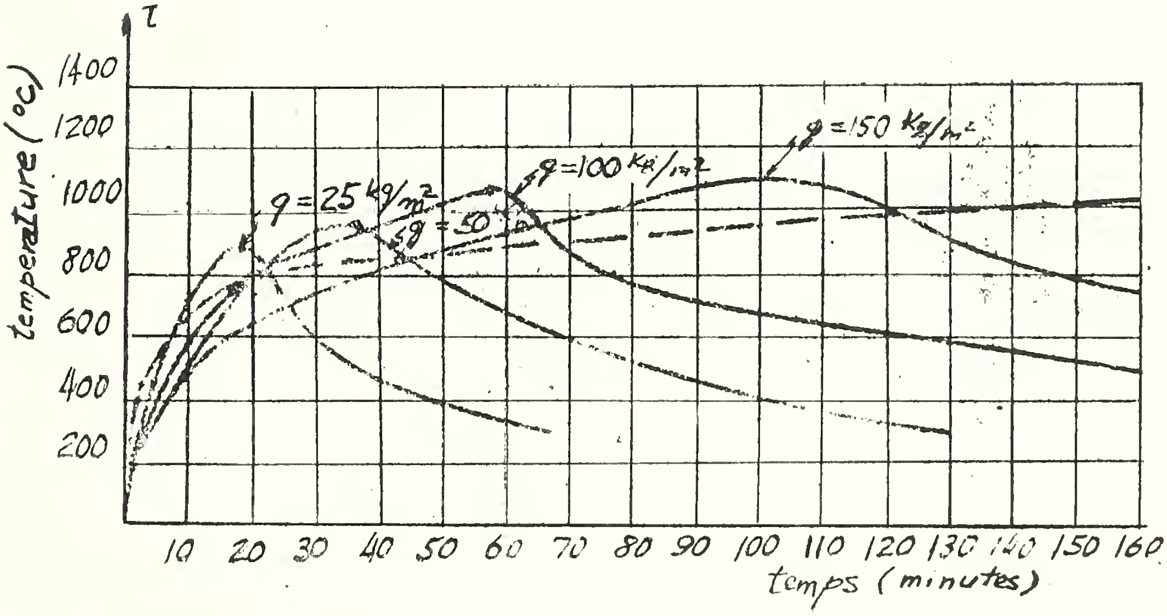

Figure 2. Standard and experimental temperature-time curves.

- - - - standard curve

———— experimental curves

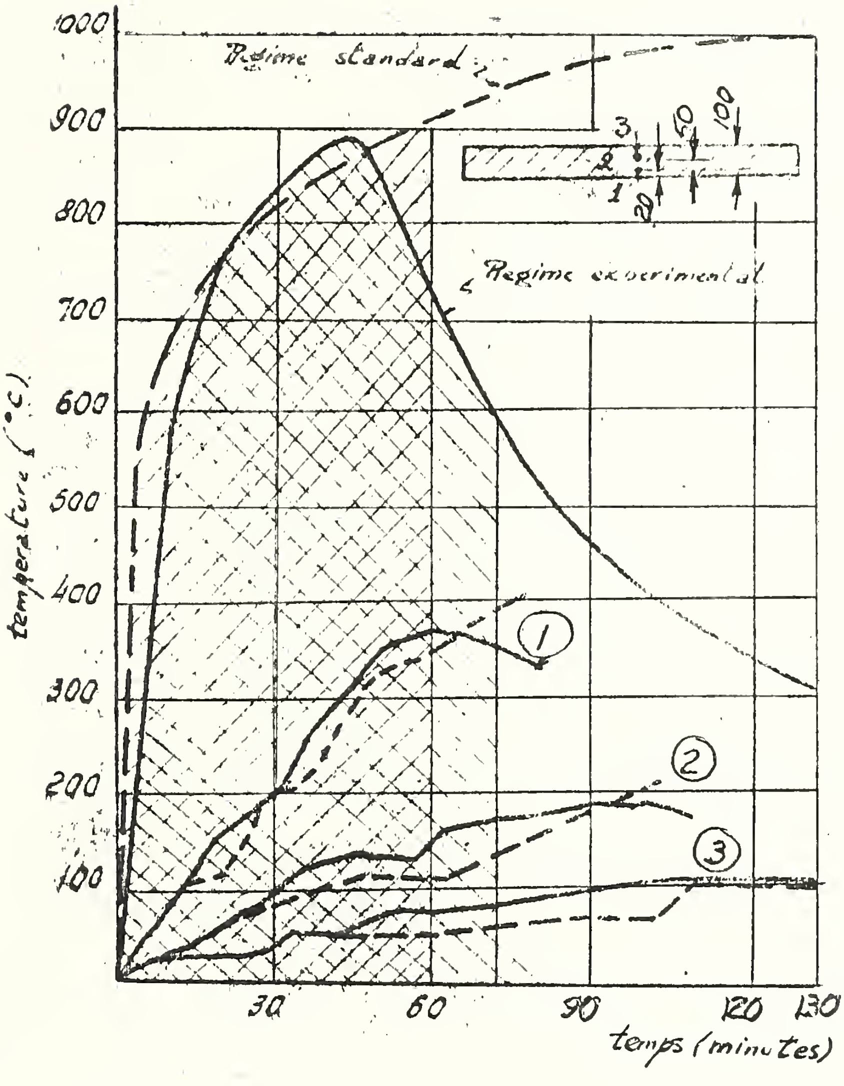

Figure 3. Temperature variations at various depths within a construction element exposed to both standard and experimental tests.

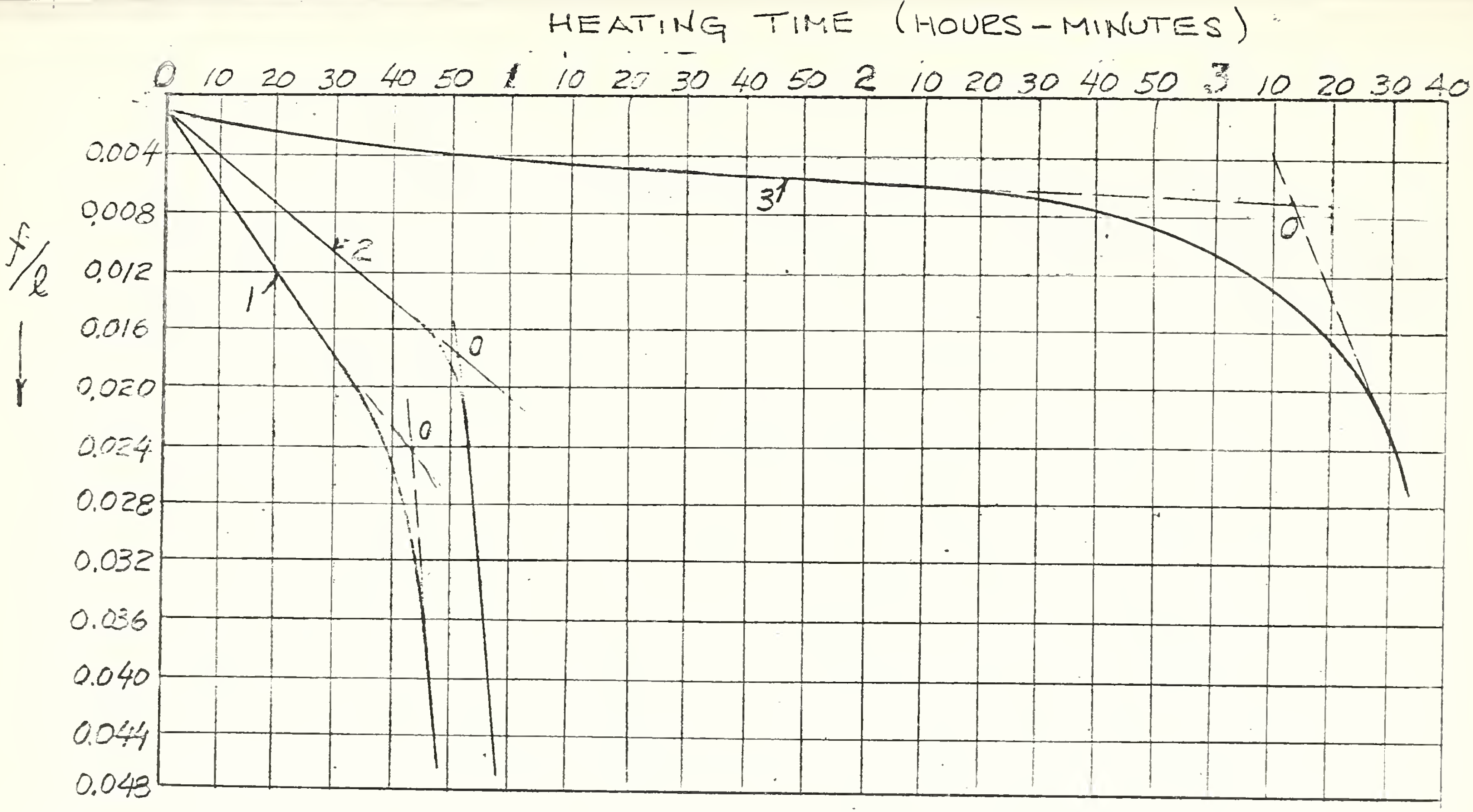

Figure 4. Deflection of structures during tests.

1. Ribbed Panel, prestressed HR steel wires, simply supported (span 3.1 m. cover of steel 20 mm, $\sigma_z/\sigma_a = 2.25$, $\sigma_z = 16,000$ kg/cm^2)
2. Ordinary reinforced concrete beam simply supported (section 180 x 360 mm, span 6.0 m, cover of steel 21 mm, two No. 3 mild steel reinforcing bars, $\sigma_z/\sigma_a = 1.96$)
3. Ordinary reinforced concrete beam fitted between and restrained by supports (section 180 x 360 mm, span 6.0 m, cover of steel 25 mm, No. 3 mild steel reinforcing bars, $\sigma_z/\sigma_a = 1.96$)

where: σ_a = working stress in the steel bar

σ_z ultimate, or yield strength of the steel bar

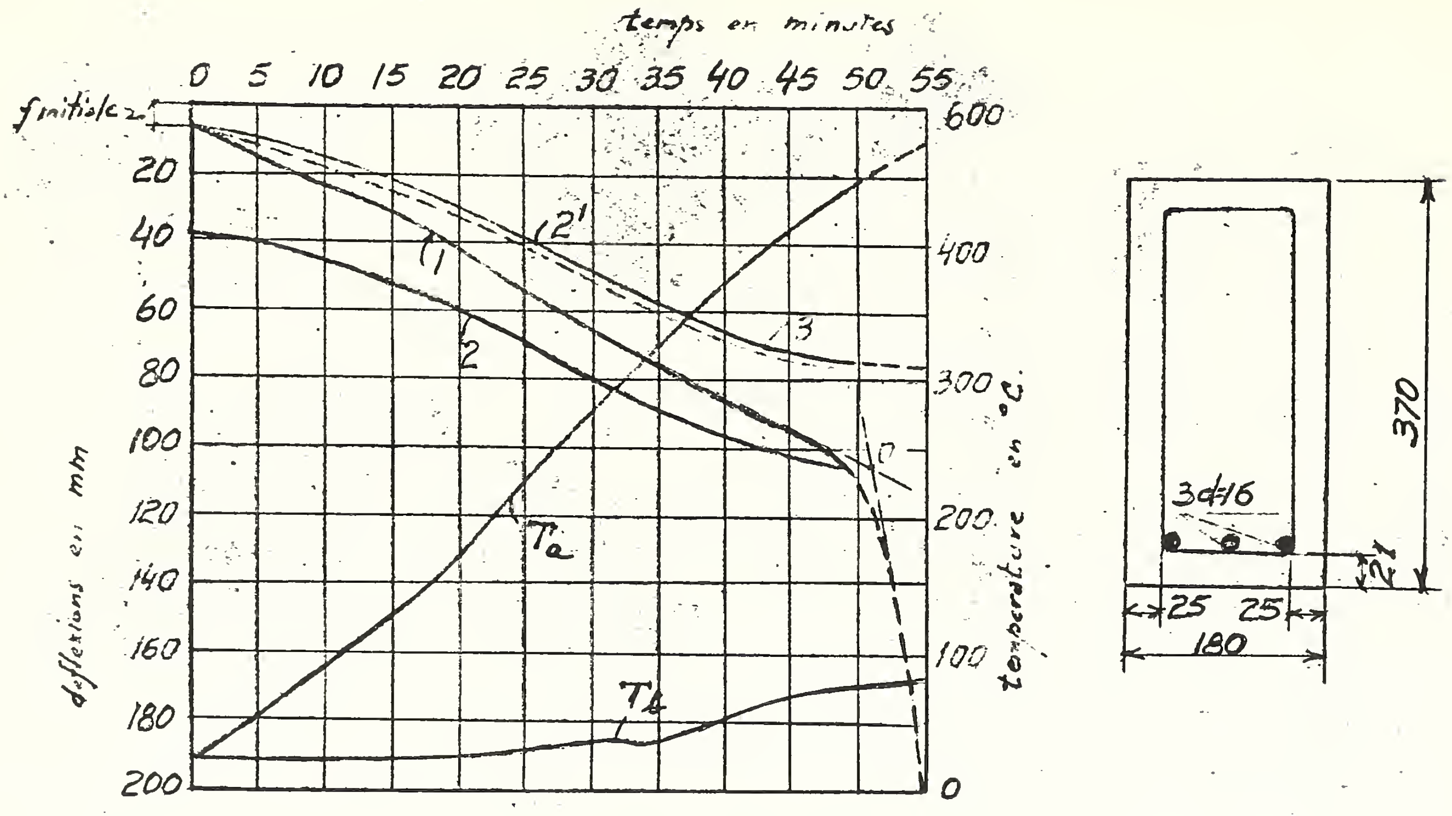

Figure 5. Deflection curves and temperatures of a simply supported beam (span 6.00 m, $\sigma_z/\sigma_a = 1.6$)

1. Deflections during heating
2. Deflections during cooling
2'. Reversible deflections during heating
3. Reversible thermal deflections calculated by the formula:

$$f = \frac{(\alpha_a T_a - \alpha_e T_e)\ l^2}{8\ h_o} + f \text{ initial}$$

where: T_a = temperature of reinforcement
T_e = temperature of concrete

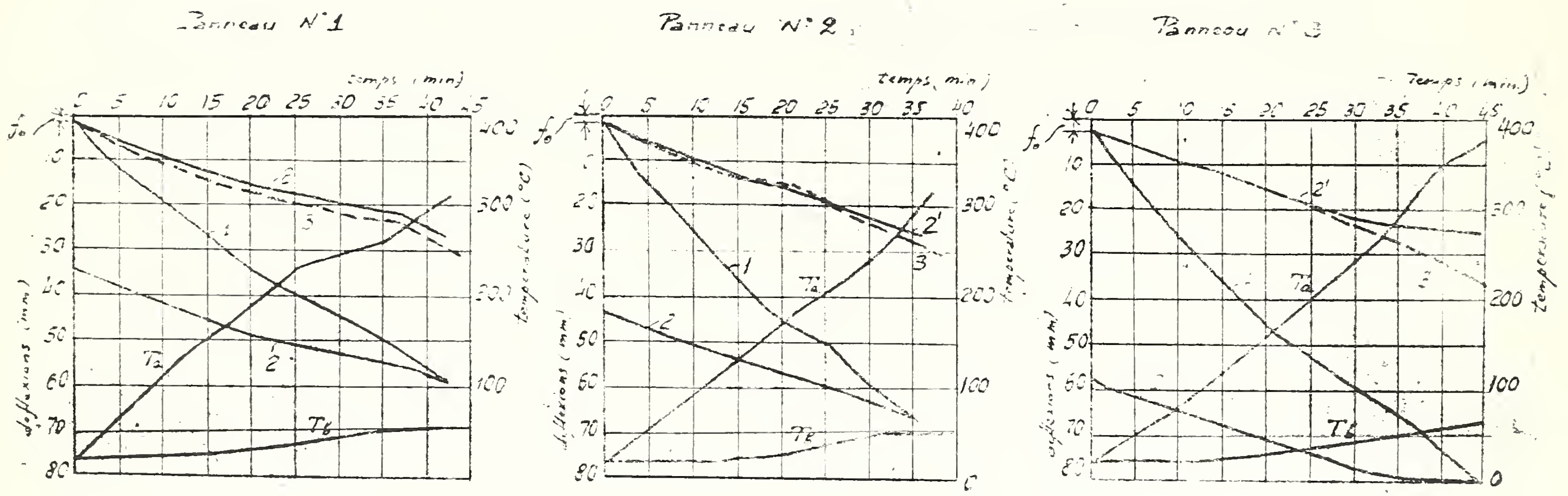

Figure 6. Deflection curves and temperatures of prestressed ribbed panels (length 3.1 m, cover 27 mm, h 19 cm, $\sigma_z/\sigma_a = 2.25$, $\sigma_z = 18{,}000$ kg/cm^2

1. Deflections during heating
2. Deflections during cooling
2'. Reversible deflections during heating
3. Reversible thermal deflections calculated by the formula:

$$f = \frac{(\alpha_a T_a - \alpha_1 T_1)\, l^2}{8\, h_o} + f_o$$

where: T_a = Temperature of reinforcement
T_1 = Temperature of concrete

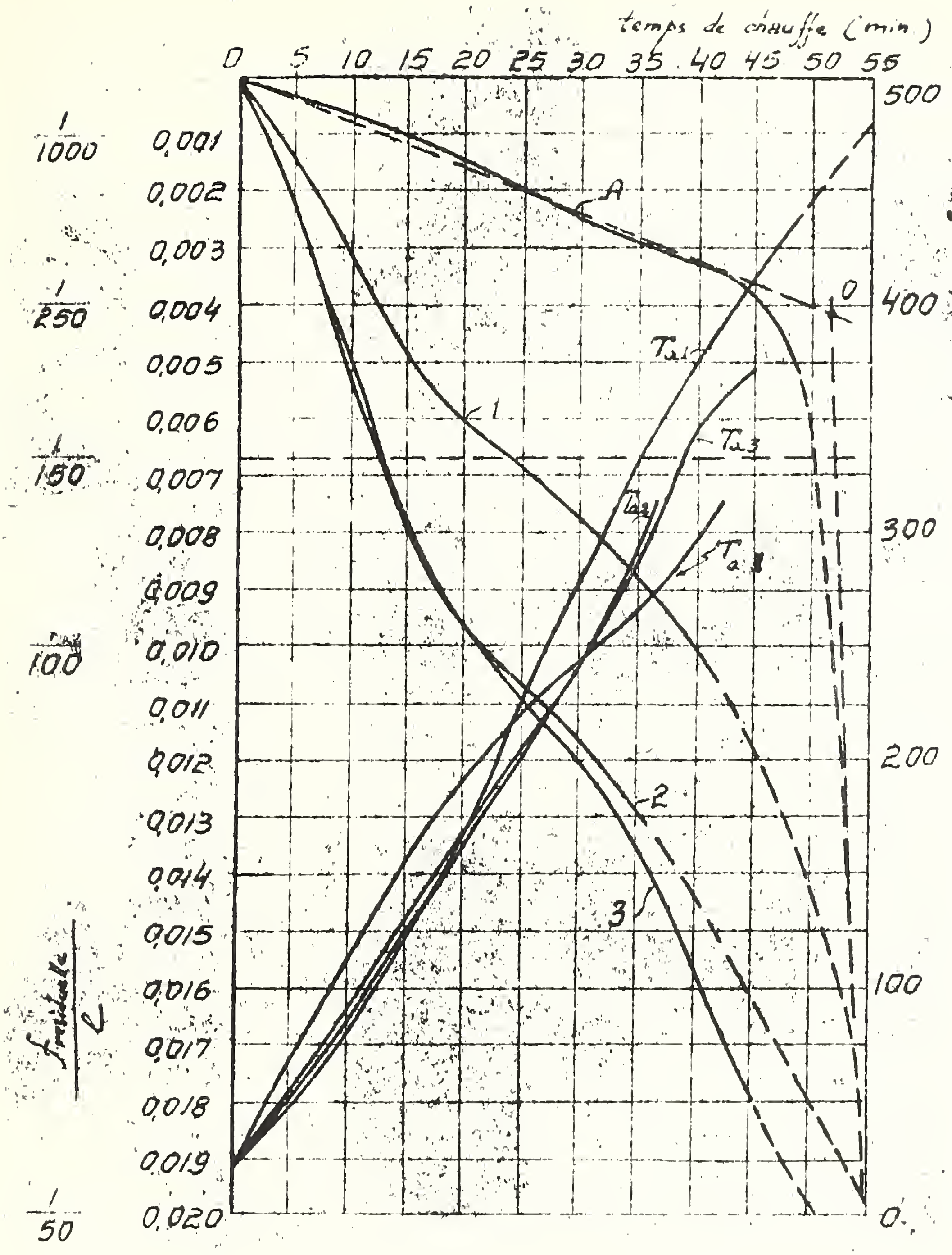

Figure 7. Redidual deflections of reinforced concrete constructions as a function of heating time and of the temperature of the steel

A - Concrete beam reinforced with mild steel

1, 2, 3 - Ribbed prestressed panels with HR steel of 18,000 kg/cm^2, strength simply supported

Ta - Temperature of reinforcement

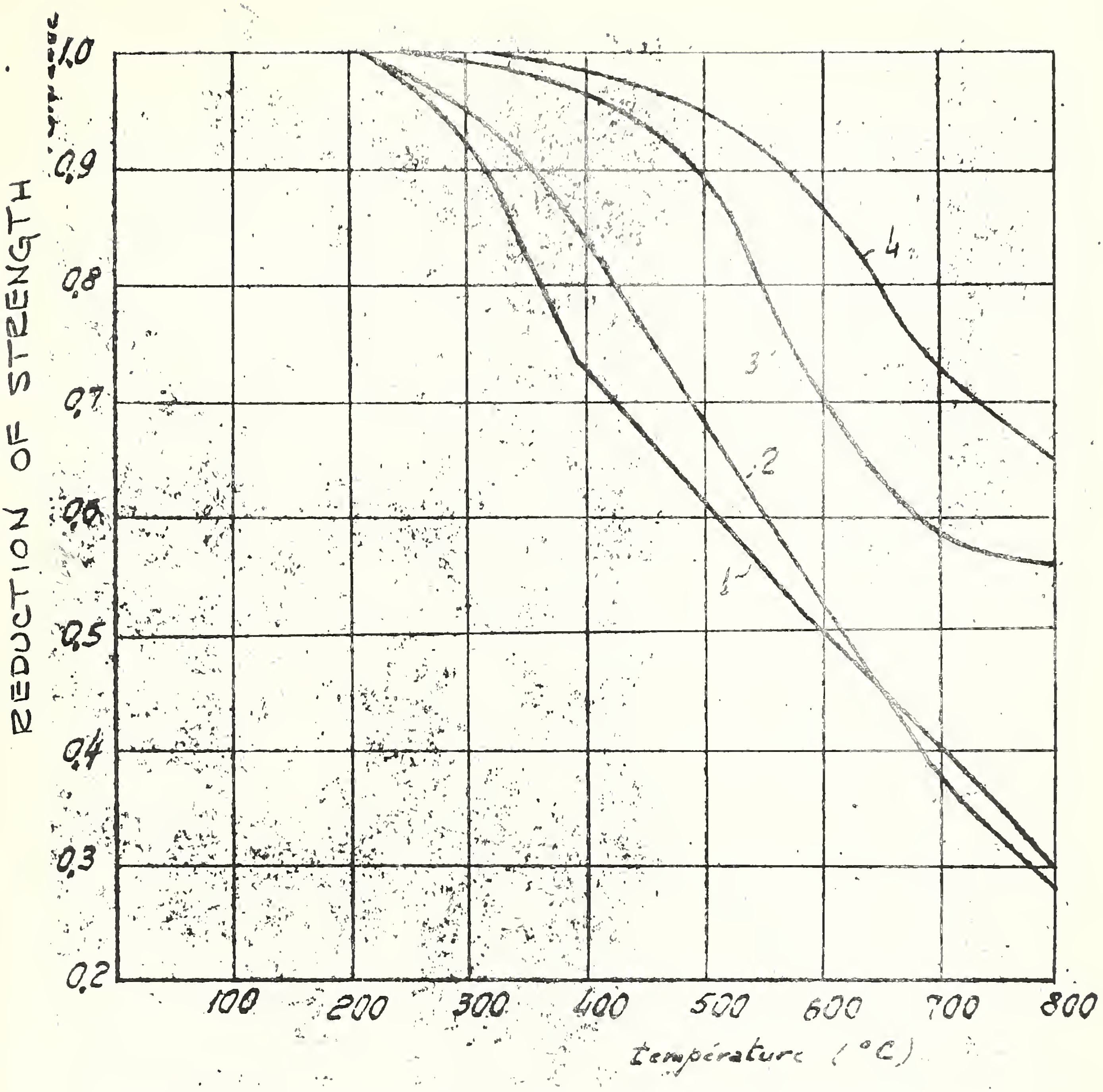

Figure 8. Loss of work hardened strength as a function of temperature

1. HR steel wire of 20,500 kg/cm^2 strength d = 2.8 mm

2. HR steel wire of 18,000 kg/cm^2 strength d = 2.5 mm

3. Cold drawn steel wire of 6,000 kg/cm^3 strength d = 5.0 mm

4. Cold deformed bar d = 12 mm of 4,900 kg/cm^2 yield strength

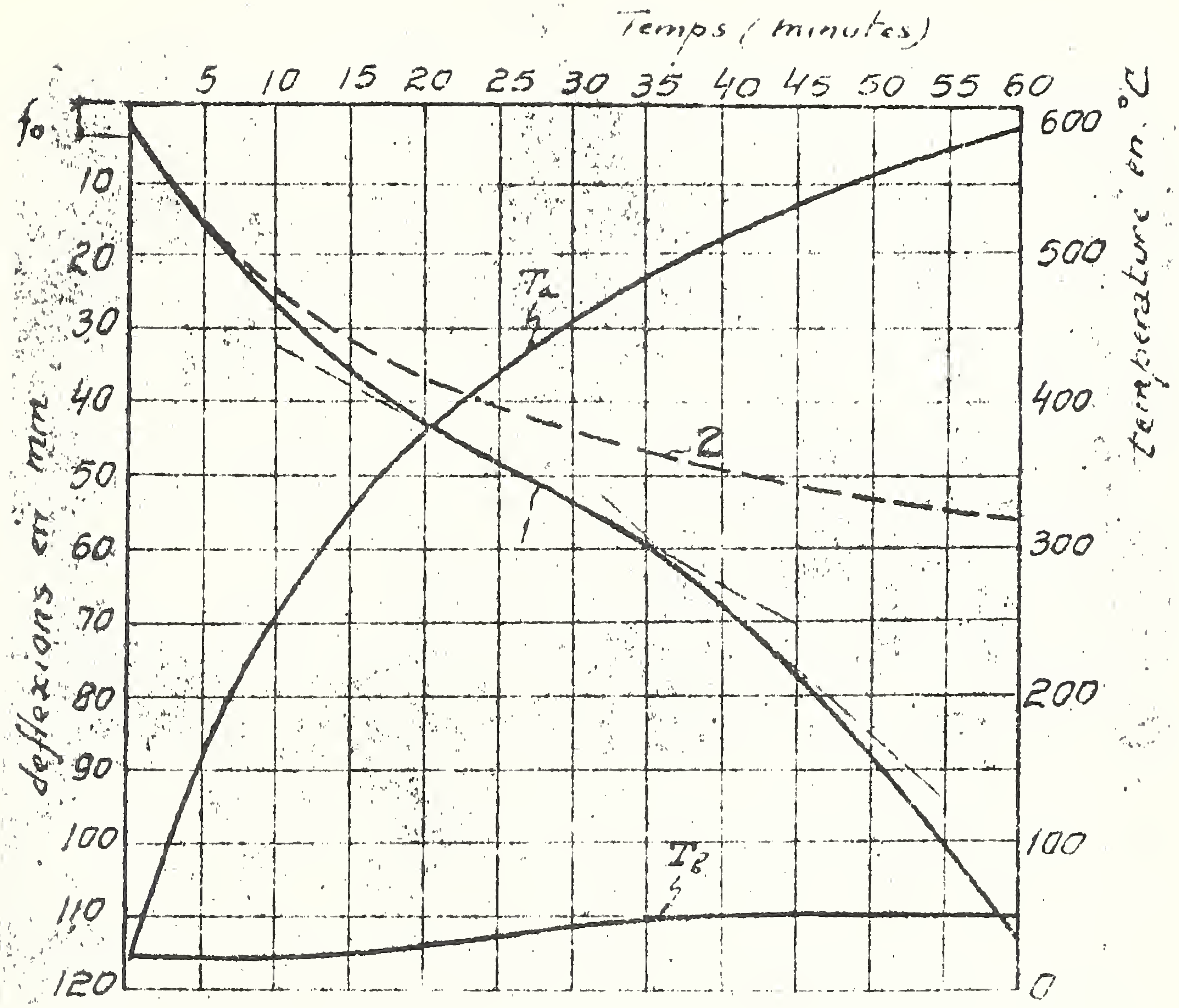

Figure 9. Deflections and temperature of a ribbed panel simply supported reinforced with welded wire fabric
(span 3.10 m, cover 10 mm $\sigma_z/\sigma_a = 1.7$ $\sigma_z = 6000$ kg/cm^2)

1. Total deflection

2. Reversible thermal deflections

$$f_{th} = \frac{(\alpha_a T_a - \alpha_b T_b)}{8 h_o} l^2 + f_o$$

Ta - Temperature of reinforcement
Tb - Temperature of compressed concrete

U.S. DEPARTMENT OF COMMERCE
Frederick H. Mueller, Secretary

NATIONAL BUREAU OF STANDARDS
A. V. Astin, *Director*

THE NATIONAL BUREAU OF STANDARDS

The scope of activities of the National Bureau of Standards at its headquarters in Washington, D.C., and its major laboratories in Boulder, Colorado, is suggested in the following listing of the divisions and sections engaged in technical work. In general, each section carries out specialized research, development, and engineering in the field indicated by its title. A brief description of the activities, and of the resultant publications, appears on the inside of the front cover.

WASHINGTON, D.C.

Electricity and Electronics. Resistance and Reactance. Electron Devices. Electrical Instruments. Magnetic Measurements. Dielectrics. Engineering Electronics. Electronic Instrumentation. Electrochemistry.

Optics and Metrology. Photometry and Colorimetry. Optical Instruments. Photographic Technology. Length. Engineering Metrology.

Heat. Temperature Physics. Thermodynamics. Cryogenic Physics. Rheology. Engine Fuels. Free Radicals Research.

Atomic and Radiation Physics. Spectroscopy. Radiometry. Mass Spectrometry. Solid State Physics. Electron Physics. Atomic Physics. Neutron Physics. Radiation Theory. Radioactivity. X-rays. High Energy Radiation. Nucleonic Instrumentation. Radiological Equipment.

Chemistry. Organic Coatings. Surface Chemistry. Organic Chemistry. Analytical Chemistry. Inorganic Chemistry. Electrodeposition. Molecular Structure and Properties of Gases. Physical Chemistry. Thermochemistry. Spectrochemistry. Pure Substances.

Mechanics. Sound. Mechanical Instruments. Fluid Mechanics. Engineering Mechanics. Mass and Scale. Capacity, Density, and Fluid Meters. Combustion Controls.

Organic and Fibrous Materials. Rubber. Textiles. Paper. Leather. Testing and Specifications. Polymer Structure. Plastics. Dental Research.

Metallurgy. Thermal Metallurgy. Chemical Metallurgy. Mechanical Metallurgy. Corrosion. Metal Physics.

Mineral Products. Engineering Ceramics. Glass. Refractories. Enameled Metals. Concreting Materials. Constitution and Microstructure.

Building Technology. Structural Engineering. Fire Protection. Air Conditioning, Heating, and Refrigeration. Floor, Roof, and Wall Coverings. Codes and Safety Standards. Heat Transfer.

Applied Mathematics. Numerical Analysis. Computation. Statistical Engineering. Mathematical Physics.

Data Processing Systems. SEAC Engineering Group. Components and Techniques. Digital Circuitry. Digital Systems. Analog Systems. Application Engineering.

• Office of Basic Instrumentation. • Office of Weights and Measures.

BOULDER, COLORADO

Cryogenic Engineering. Cryogenic Equipment. Cryogenic Processes. Properties of Materials. Gas Liquefaction.

Radio Propagation Physics. Upper Atmosphere Research. Ionospheric Research. Regular Propagation Services. Sun-Earth Relationships. VHF Research. Radio Warning Services. Airglow and Aurora. Radio Astronomy and Arctic Propagation.

Radio Propagation Engineering. Data Reduction Instrumentation. Modulation Systems. Radio Noise. Tropospheric Measurements. Tropospheric Analysis. Radio Systems Application Engineering. Radio-Meteorology. Lower Atmosphere Physics.

Radio Standards. High Frequency Electrical Standards. Radio Broadcast Service. High Frequency Impedance Standards. Electronic Calibration Center. Microwave Physics. Microwave Circuit Standards.

Radio Communication and Systems. Low Frequency and Very Low Frequency Research. High Frequency and Very High Frequency Research. Ultra High Frequency and Super High Frequency Research. Modulation Research. Antenna Research. Navigation Systems. Systems Analysis. Field Operations.

NBS